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Lateral cyclic pile-soil interaction studies on a rigid model monopile

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Abstract: The monopile foundation of an offshore wind turbine will unavoidably suffer from long-term cyclic loading during its lifetime, due to impacts from waves and wind. In this paper, a series of tests on a rigid model monopile, subjected to lateral cyclic loading, were carried out in Qiantang River silt to investigate the pile-soil interaction mechanism and accumulated deformation. The tests revealed that the accumulated displacement was closely related to the cyclic load ratio but has no obvious relationship with the relative density of soil. In contrast, the unloading stiffness is independent of the cyclic ratio but the relative density of soil.

The soil around the rigid monopile under cyclic loading undergoes a shearing stage during the first 10 cycles, followed by densification. The shearing stage dominates the cyclic responses of the rigid monopile, within which the total displacement in each cycle reduced obviously; the proportion of the elastic displacement to the total displacement for each cycle increases from ~0.5 to 0.95, and the soil pressures degrade to a large extent.

Keywords: Monopile; cyclic loading; pile-soil interaction; accumulated deformation.

1 Introduction

Currently, monopiles with a diameter of 3-8 m are widely used in European and Chinese offshore wind farms due to their relatively easier installation and lower cost. The monopile foundations of offshore wind turbines are generally short and rigid and are becoming an ideal type of foundation for offshore wind turbines located on a seabed with silt or silty sand, which is the case across much of the coastal regions of China.

During its lifetime, the monopile foundation of an offshore wind turbine is unavoidably subject to long-term cyclic loads, originating from waves and wind. This leads to accumulated rotation and changes in the stiffness of the monopile and seriously impacts the normal operation of the offshore wind turbine. Therefore, it is important to predict the deformation behaviours of the monopile under such conditions. Numerical analysis and model or field tests are generally the two most favourable ways used to solve the problem. Lesny & Hinz (2007) and Achmus *et al.* (2009) proposed new approaches to calculate the long-term cyclic behaviours of the monopile in sand while also considering the cyclic responses of the soil. They also combined triaxial test results of the soil with numerical calculations. These numerical analyses still need further verification using model or field tests.

At present, it is both challenging and uneconomical to carry out full-scale model tests or field tests on a monopile with a large diameter, thus model tests are commonly used. Le Blanc *et al.* (2010) conducted a series of model tests at 1 g on a model

monopile subjected to cyclic loading in sand with two different relative densities of 4% and 38%. In this study, approaches to predict the lateral accumulated rotation under long-term cyclic loading and the changes in unloading stiffness of a monopile foundation were presented for the first time. Roesen *et al.* (2011) carried out a series of one-way cyclic loading tests on a pile embedded in saturated sand with a relative density of 78% to 87%. They concluded that the accumulated rotation of the pile would stabilise after approximately 15,000 load cycles. Peng *et al.* (2006) and Peralta & Achmus (2010) also conducted studies on the cyclic response of a monopile placed in sand. In these experimental studies, test data were fitted to obtain the development of pile displacement and unloading stiffness; which unfortunately are unable to fully explain the mechanisms behind the accumulated deformation of a monopile. It is worthwhile to mention that Cuéllar *et al.* (2009, 2012) carried out a series of enlightening model tests to observe the behaviours of sand particles around a pile subjected to long-term lateral cyclic loading by staining part of sand particles. The study provides a detailed process of sand densification and convective cell flow of sand grains during cyclic loading, as well as the characteristics and mechanisms behind the pile foundation's accumulative deformation under long-term cyclic loading.

Most of these studies have focused on the deformation behaviours of a pile subjected to cyclic loading (including calculation formula). The pile-soil interaction has not received adequate attention, but it is important and merits further study. In this

paper, responses of a rigid monopile under long-term cyclic loading are determined experimentally, with a particular focus on the characteristics of pile-soil interaction and the mechanisms of accumulated deformation. Model tests were developed and conducted at 1 g on a model monopile subjected to lateral cyclic loading in Qiantang River silt with two different relative densities of 88% and 70%.

2 Test program

2.1 Test soil and preparation

The model tests on the monopile were carried out in a barrel-shaped soil tank, 3.70 m in diameter and 1.70 m in depth, located at Zhejiang University. The schematic setup of current model tests is shown in Fig. 1.

The soil (referred to as Qiantang River Silt in this paper) used in the model tests was taken from an excavation pit near the Qiantang River in the city of Hangzhou. It consisted of 12% sand, 80% silt and less than 5% clay particles (Jia *et al.* 2009). The properties of the natural Qiantang River silt are shown in Table 1. Laboratory tests indicated the Qiantang river silt soil has an optimum water content of 18% (ASTM D 698), a minimum dry density of 12.35 kN/m^3 (ASTM D 4254), and a maximum dry density of 15.39 kN/m^3 (ASTM D 4253). By mass controlling, the soil was poured into the tank layer by layer. Each layer was compacted to a thickness of 0.05 m using an NZH-type vibration machine. Before the filling of a new layer, a less than 5% difference in soil density ($\sim 17.3 \text{ kN/m}^3$ with $D_r = 88\%$) was ensured (by measuring densities of three randomly selected points in the current soil layer), and its upper

surface was roughed. The compaction energy was doubled to achieve a relative density $D_r = 88\%$ against $D_r = 70\%$. The properties of the tested Qiantang River silt are listed in Table 2 for a D_r of 70% and 88%.

Once the filling process was complete, a suspended water tank was connected to the pipe network set at the bottom of soil tank. The water head difference allows the water ‘flow’ into the soil tank. The soil was subjected to saturation for ~ 15 days until it reaches 95% degree of saturation, which was measured using time domain reflectometry (TDR) technology (Chen *et al.*, 2009). The water level was held at ~ 0.02 m above the soil surface during the tests.

2.2 Model pile

Le Blanc *et al.* (2010) derived dimensionless equations to predict the behaviours of a field pile in cohesionless soil using model pile tests at 1 g. Based on these equations, the current model tests selected a scaling ratio of 1:30 to simulate a monopile in the field with a diameter of 5 m. The model, steel pipe pile has 0.165 m in diameter, 0.003 m in wall thickness, 2 m in length, and 0.915 m in embedment depth, respectively. The pile-soil relative stiffness of $E_s L^4 / (E_p I_p)$ is 4.8 $\sim$ 388.6 (Poulos and Hull, 1989) from flexible to rigid piles. The stiffness ratio $E_s L^4 / (E_p I_p)$ of the model piles in the silt was estimated as 3.09 ($E_p I_p = 1.05 \times 10^6$ N.m²) for $E_s \approx 4.64$ MPa at $D_r = 88\%$. The model piles are grossly regarded as rigid.

As shown in Fig. 2, the pile wall was equipped with eight total pressure

transducers (TPTs) (CYG712, with measurement range of 0-200 kPa, to an accuracy of 0.2%~0.5%) and one pore water pressure sensor (PWP) (GE Druck PDCR81, with measurement range of 0-100 kPa to an accuracy of 0.2%). The TPT had a sensing face of 0.03 m in diameter. All sensors were embedded using a specially designed mounting block so they were flush with the pile wall surface (see Fig. 3). Each TPT and PWP was glued into a titanium case and placed in an acetyl copolymer mounting block (Bond *et al.*, 1991). To install the TPT or PWP, a hole was created at the intended position on the model pile and then the base was welded together with the pile. Next, boltholes were created on the base to install the sensors. The surfaces of the sensors were carefully installed so they were flush with the surface of the pile wall and the presence of the sensors would not affect the pile-soil interaction during loading. Finally, sensor wires were directed from the inside of the closed model pile to the outside. The model pile was jacked into the soil (via a long-range hydraulic jack) at a rate of 0.01 m/min. After installation, a 24-hour pause was needed before applying any loads so as to reduce the impact of pile driving on the bearing capacity and deformation of the model pile. After finishing a test, the pile was extracted, replaced with an un-instrumented 'dummy' pile and the instrumented pile was re-installed at a new location (with a distance larger than $7D$ to minimise the impact among the tested positions) for the next test.

2.3 Cyclic loading device

A mechanical load rig, originally developed by Rovere (2004), has been

successfully used in previous studies to apply cyclic loads to a model pile (Le Blanc *et al.*, 2010) and a model caisson foundation (Zhu *et al.*, 2013). In this paper, an improved loading device is presented, as shown in Fig. 4. It consists of a supporting base, a lever and a balance beam with a driving motor fixed to it. The major components and features are as follows: First, the loading device is a separate part that can be placed at random. Second, a blade on the balance beam is placed in a blade slot, which is attached to the supporting base (see Fig. 4a). The blade and the blade slot make up a specific pivot, which greatly reduces the friction between the balance beam and the supporting base, and also makes the balance beam very sensitive to an unbalanced force. Third, a moveable turning wheel is used to adjust length of the wire rope and ensures the balance beam stays horizontal even when there is accumulated rotation of the foundation. This way, the magnitude of the cyclic load remains stable.

A motor with a rotational frequency of 0.067 Hz is fixed to the end of the balance beam. The lever (with a length of l_a and a mass of m) is driven by the motor, which causes the mass M_1 (with a quality of m_1) to move in a uniform circular motion. As a result, the moment at the pivot (caused by M_1) changes periodically, and generates a sinusoidal force F . The loading device provides stable and long-term sinusoidal cyclic loading. Assembling different combinations of m_1 , m_0 and l (see Fig. 4) allows different magnitudes of cyclic loads (up to 700 N) to be applied.

2.4 Test program and measurement

Cyclic loading is characterised by the parameters ζ_b and ζ_c (Le Blanc *et al.* 2010):

$$\zeta_b = \frac{M_{\max}}{M_R} \quad \zeta_c = \frac{M_{\min}}{M_{\max}} \quad (1)$$

In which M_R is the ultimate static moment capacity of the pile, and $M_{\min}$ and $M_{\max}$ are the minimum and maximum moments in a load cycle, respectively. A laterally loaded rigid pile embedded in cohesionless soil may exhibit work behaviour, which renders difficulty in determining failure point (thus M_R) at the load-displacement curve. The M_R is taken as the moment at a pile-head displacement (at mudline) of $0.1D$ (Cuéllar, 2011). In this manner, the ultimate capacities were estimated as 778 N ($D_r = 88\%$) and 463 N (70%), respectively from the load-displacement curves of the model pile in Qiantang River silt (of Tests 7 and 8 under monotonic loading, see Fig. 5). In the current one way ($\zeta_c=0$) cyclic loading tests, the cyclic load ratios ζ_b were selected as 0.39, 0.3, 0.23 for $D_r = 88\%$ silt and 0.43, 0.34, 0.25 for $D_r = 70\%$ silt. The loading eccentricity of tests was taken as $e=6D$. The test program is listed in Table 3.

Four LVDTs (HCD 5000, with measurement range of 0-0.254 m to an accuracy of 0.2%) were installed on the pile to measure the instant horizontal displacement at heights (above the mudline) of 0.025, 0.165 ($= 1D$), 0.495 ($=3D$) and 0.99 m ($=5D$), as shown in Fig. 1. In particular in Tests 4, 5 and 6, an LVDT (1000DC-SE200, with measurement range of 0-0.0254 m to accuracy of 0.12%) was employed to measure the vertical displacement of the soil around the pile. A load cell (BK-1B, measurement range of 0-2000 N to an accuracy of 0.25%) was used to measure the applied cyclic load. The test data were continuously collected by a FLUKE data

collection system with a sampling frequency of 2 Hz.

3 Deformation behaviours of the rigid monopile and soil

3.1 Lateral displacement of the model pile

Displacement of the pile is essentially due to the deformation of the surrounding soil, which consists of plastic displacement d_p and elastic displacement d_e from a loading cycle. As shown in Fig. 6, the total displacement ($d_{Np}+d_{Ne}$) and the elastic displacement d_{Ne} of the pile generated in the N -th cycle are expressed as:

$$(d_{Np} + d_{Ne}) = D_{(N)\max} - D_{(N-1)\min} \quad (2)$$

$$d_{Ne} = D_{(N)\max} - D_{(N)\min} \quad (3)$$

where $D_{(N)\max}$ is the peak displacement of the pile in the N^{th} cycle, $D_{(N)\min}$ is the residual displacement of the pile after N cycles, and $D_{(N-1)\min}$ is the residual displacement of the pile after $(N-1)$ cycles.

3.1.1 Residual and peak displacement

The load-displacement curves from Test 1 (under cyclic loading) and Test 8 (under monotonic loading) are presented in Fig. 7. To improve clarity, the curves are only provided for the 1-15th cycles, 100-115th cycles, 1000-1015th cycles and 10000-10015th cycles. The figure indicates the lateral displacement of the pile increases with the loading cycles. The stiffness of the pile in the first cycle is slightly larger than that of the monotonic loading test.

A Matlab program was written to present the test data (collected by FLUKE data

collection system) by number of cycles rather than by time. The cyclic load had a period of 15 s and changed sinusoidally with time. Each period, the program picked up the maximum and minimum load values, the displacement, soil pressures and the pore water pressures.

Figure 8(a) and (b) shows the residual displacement D_{Nmin} and the peak displacement D_{Nmax} (of the pile at the loading point) of each cycle with the number of cycles N for Tests 1- 6. It shows a linear relationship between D_{Nmin} and $\text{Log}(N)$, and between D_{Nmax} and $\text{Log}(N)$; The first 10 cycles induce 55%-60% the lateral displacement D_{Nmax} at the 5000th cycle.

3.1.2 Accumulated rotation

The residual accumulated rotation is investigated. This is different from the studies of Le Blanc *et al.* (2010) and Zhu *et al.* (2013) on the peak accumulated rotation, and use of the ratio of $(\bar{\theta}_{Nr} - \bar{\theta}_0) / \bar{\theta}_s$ ($\bar{\theta}_s$ is the rotation induced in a static test at a load equivalent to the maximum cyclic load), for a monopile and a mono-caisson, respectively. Similar to Fig. 8(a), the non-dimensional residual accumulated rotation values (Le Blanc *et al.*, (2010)) of the model pile subjected to one-way lateral cyclic loading vary also approximately linearly with $\text{Log}(N)$:

$$\bar{\theta}_{Nr} - \bar{\theta}_0 \approx A \text{Log}(N) \quad (4)$$

In which A is a constant. As shown in Fig. 9, A is closely related to the cyclic load ratio ζ_b and seems to have no obvious relationship with the relative density of the soil.

3.1.3 Displacement of model pile in each cycle

The total displacement ($d_{Np}+d_{Ne}$) of the model pile at the loading point in each cycle of Tests 1- 6 is plotted against the number of cycles in Fig. 10. The total displacement ($d_{Np}+d_{Ne}$) reduces by more than 50% over the first 10 cycles. Subsequently, it reduces slowly and tends to stabilise. The lateral deformation of the monopile is dominated by the first 10 cycles.

Figure 11 shows the proportion ratio of the elastic displacement to the total displacement [$d_{Ne}/(d_{Np}+d_{Ne})$] for each cycle of Test 1. It starts at 0.5 (at $N=1$) and increases rapidly to 0.95 (at $N=10$). Afterwards, the value slowly increases with the number of cycles and approaches unity. The model pile mainly undergoes elastic deformation. The first 10 cycles are associated with shear deformation, afterwards additional deformation is caused by densification of the cohesionless soil surrounding the pile. The densification renders the load-displacement curve a tapering cyclic hysteresis loop (see Fig. 7).

3.2 Unloading stiffness

An unloading stiffness k ($=M/\theta$) is the ratio of the applied cyclic moment (M) at ground level to the pile rotation (θ) in a cycle of the unloading phase. It is recast into non-dimensional unloading stiffness $\tilde{k} = \frac{M}{L^3 \gamma' \sqrt{p_a}}$ by Le Blanc *et al.* (2010), in which L is penetration depth of pile, D is pile diameter, p_a is the atmospheric pressure, and γ' is effective unit weight. The non-dimensional unloading stiffness ($\tilde{k}_N$) of Tests 1- 6 was obtained and is plotted in Fig. 12, as a function of $\log(N)$. It

reduces in the first 10 cycles, and increases subsequently with the number of cycles.

For instance, the increase is 20% to 30% after 5000 cycles. The non-dimensional

unloading stiffness ($\tilde{l}_N$) of the model piles after N cycles can be approximated by

$$\tilde{l}_N = \tilde{l}_{10} \left(1 + \frac{N-10}{5000} \right)^B \quad (5)$$

where $\tilde{l}_{10}$ is the non-dimensional unloading stiffness after 10 cycles of loading. As a

result of the decline in unloading stiffness during the first 10 cycles being less than

5%, the value of $\tilde{l}_{10}$ can be simplified and approximated as $\tilde{l}_1$. The constant B was

obtained by best fit of the non-dimensional unloading stiffness in Tests 1- 6, and is

shown in Fig. 13. The constant B has the same meaning of dimensionless constant A_k

in $\tilde{l}_N = \tilde{l}_0 \left(1 + \frac{N-1}{A_k} \right)^B$ (Le Blanc *et al.* 2010). However, the following differences are

noted: (1) the current B ($= 12.5$) is 30% higher than $A_k = 8.02$; (2) A_k is independent of

both relative density and load characteristics, but the B is closely related to the soil

relative density and has no obvious relationship with the cyclic ratio ζ_b (see Fig. 13)

3.3 Vertical displacement of the soil surface in front of the pile

The vertically placed LVDT, located 0.05 m in front of the model pile, detected

the soil vertical displacement during the cyclic loading for Tests 4- 6, as shown in Fig.

14. There is an uplift of the soil during the first dozens of cycles, indicating that the

soil shearing deformation could contain soil dilatancy. Subsequently, the soil in front

of the pile tended to subside, or reduce in volume, which renders increase in

unloading stiffness (soil densification), as shown in Fig. 12.

4 Pile-soil interaction of the rigid model pile

4.1 On-pile pressures

On-pile soil pressures of the model test are measured by the eight TPTs during cyclic loading. In Test 1, for example, the soil pressures in each cycle are plotted in Fig. 15 for increasing number of cycles. As with the total displacement and unloading stiffness, the soil pressures reduce largely in the first 10 cycles.

The model pile rotates rigidly about a centre with zero displacement. The soil above the centre is subjected to passive resistance and below to active resistance. TPTs 7 and 8 are located at the side opposite the loading direction, 0.7 m and 0.78 m below the soil surface, respectively. At cyclic ratios of 0.4 and 0.3, the pressure values of TPT 7 are plotted in Fig. 16 as functions of the cyclic load. They increase with an increasing load, and TPT 7 is in the active zone (above the rotational centre). In contrast, little change in the pressures of TPT 8 with increasing load is noted, and TPT 8 must be close to the rotational centre, which is approximately 0.8 times the pile embedment.

The pore water pressure transducer PWP was located at 0.215 m below the soil surface at which the hydrostatic pressure was 2.3 kPa [= (0.215+0.02)×9.8]. Fig. 17 shows the measured water pressure of PWP in Test 1 during cyclic loading. The pore water pressure increases and decreases sinusoidally between 0 kPa and 3 kPa. During the test, the peak pore water pressure stays at 3 kPa with ~10% fluctuation. This suggests that no excess pore-water pressure accumulated during cyclic loading at the

depth of the transducer.

4.2 Behaviours of nearby soil under cyclic loading

Cuéllar *et al.* (2009) shows that the sand surrounding a rather flexible pile undergoes densification phase and the convection-dominated phase during long-term cyclic loading. In the densification phase, grain rearrangement and reduction of inter-granular voids takes place and is characterised by a progressive subsidence of the soil surface surrounding the pile. Once the soil subsides to a rather constant depth, the second phase starts.

The surrounding soil in the current tests exhibits shearing deformation in the first 10 cycles and densification in the subsequent cycles. The associated pile response has been discussed before, as highlighted herein. During the shearing deformation, (1) the plastic displacement d_p of the soil accounts for a large proportion of the total displacement in a given cycle (see Fig. 11); (2) The unloading stiffness of the pile decreases (see Fig. 12). (3) The soil dilatancy (associated an increase in the soil volume) occurs, which, together with the lateral movement, may cause the uplift of the soil surface in front of the pile (see Fig. 14). As for the densification stage, a reduction of inter-granular voids seems to have occurred, as the elastic displacement d_e of the soil also accounts for a large proportion of the total displacement in a cycle (see Fig. 11), and the unloading stiffness increases (see Fig. 12).

5 Limitations

This paper has made a preliminary evaluation of pile-soil interaction mechanisms and accumulated deformation for a rigid model monopile under lateral cyclic loading in Qiantang River silt. There are a few limitations to this study. The model tests were conducted at 1 g condition and based on the dimensionless equations for scaling of laboratory tests. These dimensionless equations in turn were based on the ultimate limit state of the sand. Their applicability to soil at an elastic-plastic state is uncertain. The cohesionless soil exhibits dilatancy at low stress levels (e.g. in laboratory tests at 1 g) (Guo and Qin, 2010), which may not occur in the field. As a result, the cohesionless soil in model tests may be associated with a higher peak friction angle than the soil with the same relative density present in the field (Le Blanc *et al.*, 2010).

6 Conclusions

In this paper, six model tests were carried out on an instrumented rigid model pile in Qiantang River silt that was subjected to lateral cyclic loading with different cyclic ratios for 5000-10000 cycles. The main conclusions drawn are as follows:

(1) The accumulated displacement is closely related to the cyclic load ratio and has no obvious relationship with soil density. In contrast, the unloading stiffness is independent of the cyclic ratio but is related to the relative density of the soil. The peak and residual accumulated displacements increase linearly with the logarithm of the number of cycles.

(2) The rotational centre of the rigid model pile is at approximately 0.8 times of the pile embedment during cyclic loading.

(3) There was generally a slight degradation of the peak value of on-pile soil pressure in the first 10 cycles, which tends to be invariable afterwards.

(4) The first 10 cycles (of shearing stage) have a great influence on the cyclic responses of the rigid monopile under cyclic loading, rather than the subsequent densification stage.

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395

396 Notations

397

398	A	fitting parameter for non-dimensional rotation of pile
399	A_k	dimensionless constant
400	B	fitting parameter for non-dimensional unloading stiffness of pile
401	C_c	coefficient of curvature
402	C_u	uniformity coefficient
403	c'	effective cohesion
404	D	pile diameter
405	D_r	relative density
406	d_e, d_p, d_{Np}, d_{Ne}	deformation parameters
407	$D_{(N)\max}, D_{(N)\min}$	maximum and minimum deformation parameters
408	d_{50}	average grain size
409	d_s	specific gravity
410	E_c	compressive modulus
411	E_p	elastic modulus of pile
412	E_s	elastic modulus of soil
413	e	load eccentricity
414	F	force
415	g	gravitational acceleration
416	I_p	moment of inertia of pile
417	k	unloading stiffness
418	$\tilde{l}, \tilde{l}_0, \tilde{l}_1, \tilde{l}_{10}, \tilde{l}_N$	normalized unloading stiffness
419	L	embedded depth of pile
420	l, l_0, l_a, l_c	load device dimensions
421	$M, M_{\max}, M_{\min}$	moment at ground surface
422	M_R	static moment resistance of pile
423	m, m_0, m_1	load device masses
424	N	number of load cycles

425	P_a	atmospheric pressure
426	γ'	effective unit weight
427	γ_{sat}	Saturated unit weight
428	ζ_b, ζ_c	load characteristic parameters
429	w	natural water content
430	w_L	liquid limit
431	w_{op}	optimum water content
432	w_p	plastic limit
433	μ	poisson's ratio
434	ϕ'	effective internal friction angle
435	θ	pile rotation
436	θ_0	pile rotation in the first cycle
437	θ_s	static pile rotation
438	θ_{Np}	peak accumulated rotation in N th cycle
439	$\tilde{\epsilon}_{Nr}$	non-dimensional residual accumulated rotation in N th cycle
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Tables and Figures

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Table 1. Properties of natural Qiantang River silt

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Specific gravity G_s	Natural water content w (%)	Optimum water content w_{op} (%)	Plastic limit w_p (%)	Liquid limit w_L (%)	Average grain size d_{50} (mm)	Uniformity coefficient C_u	Coefficient of curvature C_c
2.69	13	18	22.6	31.7	0.0328	2.48	1.35

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Table 2. Properties of Qiantang River silt

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No.	Saturated unit weight (kN/m ³)	Relative density D_r (%)	Effective internal friction angle (°)		Effective cohesion (kPa)	Compressive modulus (MPa)	Poisson's ratio
			Peak	Residual			
1	19.1	88	41.5	35.5	0	6.25	0.28
2	18.8	70	37.4	35.5	0	5.35	0.30

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Note: The cohesion and friction angle and Poisson's ratio of the samples were obtained by triaxial tests (CD) with confining pressures of 20, 40 and 80 kPa, respectively.

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The compressive modulus was obtained through uniaxial confined compression tests with compressive stress between 0-12.5 kPa.

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Table 3. Test program

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No.	Relative density of silt D_r	Height of lateral loading e	Cyclic ratio ζ_b	Number of cycles N
Test 1	88%	$6D$	0.39	10015
Test 2	88%	$6D$	0.30	5000
Test 3	88%	$6D$	0.23	5000
Test 4	70%	$6D$	0.43	5000
Test 5	70%	$6D$	0.34	5000
Test 6	70%	$6D$	0.25	5000
Test 7	70%	$6D$	Monotonic	—
Test 8	88%	$6D$	Monotonic	—

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Fig. 1. Schematic setup of the model tests (unit: mm)

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(a) Schematic diagram (Unit: mm)

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(b) Physical diagram

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Fig. 2. Model pile and on-pile transducers

(a) TPT

(b) PWP

Fig. 3. Connection between transducers and model pile

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(a) Schematic diagram

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(b) Physical diagram

Fig. 4. The cyclic loading device

Fig. 5. Load-displacement curves of Tests 7 and 8

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Fig. 6. Schematic drawing of the lateral displacement of the pile

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Fig. 7. Load-displacement curves of Tests 1 and 8

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(a) Residual accumulated displacement

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(b) Peak accumulated displacement

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Fig. 8. Measured residual and peak accumulated displacement of the pile

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Fig. 9. Values of A

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Fig. 10. Measured total displacement of the pile in each cycle

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543 Fig. 11. Proportion of elastic displacement to total displacement in each cycle (Test 1)

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(a) $D_r=88\%$

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(b) $D_r=70\%$

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Fig. 12. Non-dimensional unloading stiffness of the pile

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Fig. 13. Values of B

Fig. 14. Vertical displacement of the soil surface in front of the pile

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(a) TPT 1-TPT 4

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(b) TPT 5-TPT 8

565 Fig. 15. Measured on-pile soil pressures as a function of number of cycles (Test 1)

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(a) $D_r=88\%$

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(b) $D_r=70\%$

Fig. 16. Measured on-pile soil pressures of TPTs 7 and 8

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Fig. 17. Measured pore water pressures (Test 1)